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HOLT-McDERMOTT MINE VISIT: STABILITY ASSESSMENT OF SHALLOW UNDERGROUND OPENINGS

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Canadian Mine Technology Laboratory

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HOLT-McDERMOTT MINE VISIT: STABILITY ASSESSMENT OF SHALLOW UNDERGROUND OPENINGS

by

M.C. Bétournay*

ABSTRACT

In May, 1988, a site survey was carried out at the Holt-McDermott mine for the purpose of examining the stability of shallow underground openings. The mine, which at this time has not begun the process of designing the surface crown pillars, will begin its mine activity by creating the surface crown pillars.

Various observations and calculations are made on the rock mass, soil, stress and mining conditions. The mass is separated into geomechanical sectors and surface crown pillar failure modes are examined. Various design elements for the pillars are presented.

KEYWORDS: surface crown pillar, geomechanical sectors, open stope, failure mechanisms, rock mass quality.

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VISITE DE LA MINE HOLT-McDERMOTT: ÉVALUATION DE LA STABILITÉ DES OUVERTURES SOUTERRAINES PEU PROFONDES

par

M.C. Bétournay*

RÉSUMÉ

En mai, 1988, un examen du site de la mine Holt-McDermott a été fait dans le but d'examiner la stabilité des ouvertures situées près de la surface. La mine, qui jusqu'à date n'a pas entammé le processus de conception des piliers de surface, débutera son activité minière avec la création des piliers de surface.

Divers observations et calculs sont faits pour le massif rocheux, le mort-terrain, les contraintes et les conditions d'abattage. Le massif est séparé en secteurs géomécaniques et des modes de ruptures des piliers de surface sont examinés. Des éléments de conception pour les piliers sont inclus.

MOTS CLÉS: pilier de surface, secteurs géomécaniques, chantier ouvert, mécanisme de rupture, qualité du massif rocheux.

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1. INTRODUCTION

A cooperative, shared cost ground investigation between American Barrick Resources Corporation and CANMET/MRL has been carried out at the Hold-McDermott Mine, situated in Northeastern Ontario, near the Quebec border, Figure 1.

The study was carried out within the general framework set for CANMET/MRL's involvement in cooperative research. Accordingly, the non-consultant, research role CANMET/MRL carried out, has the following objectives:

- to establish applied rock mechanics reference data which can be reviewed by the company and consultants retained by the company for making mine planning and operational decisions, providing that the study results are considered by the company and consultant as pertinent.

- to develop new applied investigational and analytical methods and monitoring equipment suited to the specific requirements of the study and for general use by the industry.

- to transfer the latest and most up-to-date developments in rock mechanics technology relevant to the specific requirement of the study.

Discussion with mine personnel and underground visits were carried out on May 15 and 16, 1988.

2. MINING BACKGROUND

The mine extracts gold which is finely disseminated in massive basalt. The ore is believed to have been emplaced as a result of, and in the footwall of, the McKenna fault. This major geological feature strikes

E-W and usually dips 70° to the south.

The extent of the mineralization is variable, Figure 2. Laterally, it extends 3 to 10 m from the fault and longitudinally it is lenz shaped.

Four large mining blocks are planned near surface, Figure 3. These blocks will be excavated 50 m high and left open. However, only two will have veritable surface crown pillars, the 2-1-0997 block, 40-60 m below overburden, and the 3-1-1015 block, 60-80 m below overburden.

The mine will use a long hole mining method with 5.36 cm holes in a transversal ring pattern. Ring patterns are repeated longitudinally every meter.

So far, the mine has created the 100, 150, 200, 250, 300 and 350 m ore zone drifts and drawpoints as well as footwall haulageways. Production will begin shortly.

3. ROCK MASS CONDITIONS

The main elements of the rock mass which can potentially affect opening and surface crown pillar stability are:

- major site-wide discontinuities;
- joints;
- weak rock units;
- water inflow.

3.1 Major Discontinuities

Several through-going discontinuities occur at this mine, all classified as faults.

The main fault, referred to as the McKenna Break, forms the hanging wall boundary of the mineralization. The break is characterized by the presence of 2-5 cm of gouge. Other structural features, found in the footwall, intersect but terminate at the main fault. This is the case for joints and several cross faults, the latter intersecting at approximately 30° and oriented NE-SW, vertically dipping, Figure 4.

Joints and cross faults are often calcite filled. The cross faults have immediate walls that are schistose.

3.2 Joints

The rock mass is dissected by a number of joint families. The most pervasive strike north, with a high to vertical dip. These were seen in the footwall and the immediate hangingwall. They are long, > 2 m, spaced 30 cm-2 m, planar and often filled with a thin calcite coating.

At least two other families are commonly found. One is a shallow west dipping (20°-40°) joint; it is planar, > 1 m length and spaced 10-20 cm, calcite also appearing in the joint. The other is a vertical joint striking E-W, also planar.

There were other, occasionally occuring, joints, with the following strikes and dips: 60,70 SE and 340,70 NE, but these rarely contribute to ground instabilities.

The joints intersect each other in a throughgoing way. Often, several families occur simultaneously, the intersections form blocks 0.3 -1 m^3 in size.

3.3 Weak Rock Units

Located in the footwall of the orebody is a major zone of graphite schist. Figures 2 and 4 show the rock unit to be of variable

width and extent. It can adjoin the orebody, but usually is more than 5 m from it. It takes on a branching lenzoid shape, overlapping horizontally and vertically. The schistocity is parallel to the orientation and dip of the zone and the fault.

3.4 Water Inflow

The mine has encountered serious water inflow underground. The water is believed to originate with water-laden overburden and carried into the rock mass by numerous diamond drill holes drilled from surface. A number of these pass through the orebody from hangingwall to footwall. At the time of the visit, none of these holes had been grouted, casings had been removed leaving the holes open to water-bearing overburden.

The water is believed to flow into the faults and joints from the boreholes. Underground exploration holes and advancing mine development have intersected severe water flow. Pressures of up to 160 psi have been encountered at the 100 m level, and 400 psi at 300 m level, indicating hydrostatic pressure (1).

On this site visit several examples of this condition were seen. Water was entering mine openings through the cross faults, washing away and softening the calcite infilling. At various locations (bored ventilation raise, ~ 80 m depth and below; drifts) water enters via the joints.

4. GROUND CONTROL

There is no apparent large scale ground control problem at the mine. The mechanical anchor bolts are sufficient to presently maintain opening integrity.

On a localized scale in drifts, the graphite schist area can require screening as adjoint support.

The cross faults present little control problems, even at the intersection of the main fault trace. Minor ravelling problems occur on the immediate contact of these faults.

Large scales excavation will generate more serious ground control problems. The observations presented here are related solely to effects of development work performed so far.

5. ROCK MASS CHARACTERISTICS

The basaltic rock of the Holt-McDermott mine, though variable in degree of silicification (a result of fault action) or chloritization, is basically a sound rock with no anisotropy. Laboratory strength tests (2) indicate a range of 74 to 277 MPa unconfined compressive strength, 14 to 44 MPa tensile strength, for intact rock behaviour. The modal value is near the upper end of each range.

There are subtypes, such as highly chloritized or carbonate filled rock, which are much weaker.

The graphite schist rock is quite variable in mineralogy, possessing a range of strength values, which at their best do not exceed 60 MPa unconfined compressive strength. This zone should be considered weak, especially to tensile stresses. Any failure testing of this material must take into consideration the material variability and anisotropy (e.g. schistocity) with regards to loading directions.

The main ore zone rock type test results are inconclusive.

Though joint spacing and length varies for each family there are distinct geomechanical sectors, each defined by different combinations of RQD and mass behaviour, Figure 5.

The weakest sector is the graphite schist, estimated to have low RQD values, no better than 50%, with a disintegrating behaviour.

The immediate hangingwall zone (up to 5 m into the hanging wall from the McKenna fault) and the footwall sector (between the edge of mineralization and the graphite zone) possess RQD values from 60 to 70% and most often ravel as a result of effective, dense, pervasive jointing.

The fault trace forming the hanging wall side of the orebody drifts is planar and very stable.

The ore zone appears to be a good geomechanical sector, RQD = 85-95%, the mass being blocky from joint intersections. The far field hangingwall zone is also more massive, RQD > 85%.

Using the NGI rock mass characterization system †(3) and the preliminary site observations gathered on this visit, the rock mass quality assessment for these sectors (around near surface openings) is:

1) Graphite zone

$$Q = \frac{30}{9} \times \frac{2.0}{4.0} \times \frac{0.5^*}{2.5} = 0.33$$

2) Immediate hangingwall and footwall

$$Q = \frac{65}{9} \times \frac{1.0}{2.0} \times \frac{0.5^*}{2.0} = 0.83$$

3) Ore zone

$$Q = \frac{90}{6} \times \frac{1.0}{2.0} \times \frac{0.5^*}{2.0} = 0.94$$

4) Far field hangingwall

$$Q = \frac{85}{4} \times \frac{1.0}{2.0} \times \frac{0.5^*}{2.0} = 2.66$$

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† Under this system, six parameters are considered for rock mass quality:

$$Q = \frac{RQD}{Jn} \cdot \frac{Jr}{Ja} \cdot \frac{Jw}{SRF}$$

The original reference prescribes values for each of these parameters. Jn relates to the number of joints families, Jr the joint roughness, Ja the alteration of the joint, Jw the water inflow, SRF the stress conditions.

* For upper excavations < 100 m (near surface crown pillar only), water pressure < 160 psig.

Qualitative representation of the Q values against opening specifications are presented in Figure 6.

These Q values will increase when the mine has finished its grouting operation and the excavation are subjected to minor inflow only. A value of 1.0 for Jw may be used then, thereby doubling the Q value.

6. SURFACE CROWN PILLARS

The mine operators are currently addressing the design of potential surface crown pillars. The upper stopes will be excavated first, then the final surface crown pillar dimensions created and subsequently, the lower mining blocks will be mined.

This is the most demanding surface crown pillar strategy. It requires that all data gathering and analysis be completed, permanent support methods emplaced, and an active monitoring program established, before the opening is finished, and for the rest of the life of the mine. Here, the burden is on the mine to have sufficient data to have a good grasp of the situation for such a design. Dimensions and support cannot be underdesigned.

The following subjects will be covered in this preliminary assessment of the mine's surface crown pillars:

- effect of the mining method;
- support;
- failure mechanisms;
- overburden problems.

6.1 Effect Of The Mining Method

The mine is planning to use a longhole mining method, with no backfill emplaced.

Ground vibration and stress redistribution with this mining method subjects large openings to blast vibrations which will easily damage the surrounding rock mass. The immediate footwall and hanging wall, with such a low Q value and a tendency to ravel, can break up easily.

It remains to be seen how much of the peripheral rock will slough into the stope and how extensive the adjoining zone of lossening, or shattered zone will be.

Also, the stresses existing before each blast will be redistributed in a major and sudden fashion causing sudden strain energy imposition and, because the rock mass can deform readily, lead to further ground breakage. These situations can be detrimental to opening and surface crown pillar stability.

Consequently, the pillar will have a reduced net thickness of undisturbed rock, having been fractured at its base. At the present moment, the shallowest opening is 40 m below the overburden. However, even the upper 3-4 m of the bedrock (at the contact with the overburden) is fractured and loosened, as seen from drill core. It is evident that in all, the surface crown pillar will have much less than 40 m of unfractured rock, existing between and upper and lower broken zone. The mine should therefore adopt a conservative value for pillar thickness, not the distance from the upper bedrock limit to the underground opening.

The mine should also design future artificial pillar support according to this situation. The support must be long enough to be anchored in rock that will be unfractured after the entire block is mined. The support also has to maintain the rock around the opening in a tight configuration to prevent raveling and ground deterioration. In this case, cable bolts, applied before stoping, is an example of required support.

6.2 Failure Mechanisms

The design of the mine's surface crown pillars should not be

restricted to the ore zone above the opened stopes. The stability of a much wider volume of rock must be taken into consideration.

For this site, application of elastic analytical formulas is questionable. Tensile and shear behaviour considerations should be adopted. Several failure mechanisms can be considered: ravelling, mass shear failures and plug failures.

Ravelling can be generalised or, as often seen when hanging wall conditions are poor, progressive chimneying along the hanging wall until the caving chokes itself of, or the overburden is reached. It is also possible that such chimneying can occur along the cross faults, infilling material and neighbouring schistocity having already been partially washed out and the weak schistocity adjoining.

If the graphite zone is close enough to the damaged zone, large scale movements of that zone towards the opening could take place. In this, as in the generalized ravelling case, the underground opening becomes much larger than planned. Therefore, stability assessment under such conditions must also be calculated.

Other failure modes merit discussion. Applying pre-existing stress values at the level of the opening, two failure mechanisms can be examined, Appendix 1: shear failures in the mass and plug-like failures between well defined major weaknesses, e.g. fault and graphite schist.

Calculations show that mass shear failures are unlikely and plug failures are possible if all pre-existing streses are considered.

But creating underground openings redistributes pre-existing stresses. A new stress field with stresses varying around the opening will be created.

In the case of the shallowest opening, application of the Geldart and Udd (4) stress redistribution equation, Appendix 2, shows that 126.4 MPa compressive stress is theoretically available at the base of the surface

crown pillar compared to 1.8 MPa strictly using the pre-existing stress value.

It is thus likely that this definite increase in compression for the rock volume around the base of the pillar contributes significantly to the stability of the pillar.

Numerical modelling techniques offer the best opportunity to obtain displacements under given stress, rock mass and stope geometry considerations. Degradation of opening periphery and displacements of the surface crown pillar are discernable. Though it does not lend itself to representation of specific failure mechanisms, it remains the best tool to evaluate rock mass stability. The small scale rock movements, block sizes, and heterogeneous rock types would preclude using either distinct elements or boundary elements. Finite element modelling would provide the most representative results.

The mine will have to examine the potential for overburden inflow into the mine if a large failure takes place. The saturated overburden, consists of upper varved clay unit, a middle silty sand layer and lower silty till.

The overburden reaches a maximum depth of 50 m over the surface crown pillars, Figure 3.

7. SURFACE CROWN PILLAR DESIGN PROCESS

The Holt-McDermott mine is ready to embark on the design of surface crown pillars for their uppermost opening. So far, the mine operators have not begun the process of designing their surface crown pillar.

At this stage, very little information has been collected specifically for the purpose of surface crown pillar design/stability. It is suggested that a design process such as the one established for surface

crown pillars by Bétournay (5), Figure 7, be adopted.

As a minimum, the following supplementary work should be performed:

Geotechnical Investigations:

- obtain sufficient joint measurements to reflect general and locally different/ critical conditions;
- obtain joint spacing orientation, persistence, termination, infilling, roughness;
- perform in-situ stress measurement, making sure the test margin of error is less than the value obtained.
- perform N size diamond drilling in the surface crown pillars, at various orientation, to characterize the rock mass according to an empirical system (e.g. NGI), identify geomechanical sectors, and perform tests (taking anisotropy into consideration) to obtain lab m & s and strength parameters.

Data Analysis:

- establish in 2-D and 3-D the distinct geomechanical sectors reflecting, joint density, RQD, rock mass classification values, etc;
- use stereonet methods (6) to verify for local block instabilities.

Design Methods:

- use appropriate design methods, that will address tensile and shear failures. Application of elastic theory formulas is inappropriate;
- numerical modelling is recommended, using a failure criterion, for each distinct geomechanical sector, that best represents near surface ground behavior.
- the layer of broken rock around the underground openings and in contact with the overburden should not be included in the design methods. The overburden should be investigated for their potential to liquify, potential to self support over a given size opening, subsidence potential.

Monitoring:

- the instrumentation should be placed before stope excavation has begun.
- wall and crown movements should be monitored. Ideally, localised progressive failure should be detected as well a generalized degradation.
- monitoring of the hanging wall and footwall should be carried out. For field movements can be detected with extensometers and fissurometers. Although the latter only provides indication of movement, it can be used as failure sensors monitor.

Remedial Measures:

- the mine should continue its efforts to substantially reduce the inflow of water around the surface crown pillar area. Rock mass damage and potential for instabilities would be reduced.

8. SUMMARIZED OBSERVATIONS

1) The Holt-McDermott mine, which will soon begin production, has several site characteristics which will influence the stability of surface crown pillars:
   - McKenna orebody fault;
   - cross faults;
   - immediate hanging wall and footwall jointing;
   - water inflow;
   - graphite schist zone.

2) The stability of the surface crown pillars at this site should be seen in the context of tensile and shear failures, not compressive failures.

3) The design of the surface crown pillars must take into consideration more than the volume of rock defined by the ore zone. At a minimum the immediate footwall, hanging wall and graphite schist zone should be included.

4) The long hole mining method the operators plan to use to excavate the

stopes will probably damage/loosen the immediate hanging wall and footwall. The overlying volume of rock in surface crown pillars must therefore be designed to take such expanded openings into consideration.

5) Some of the potential pillar failure modes are:
   - general ravelling;
   - hanging wall ravelling;
   - mass plug failure between McKenna fault and closely located graphite schist zone.

6) The mine has a number of essential design steps to complete before mining the stopes and leaving the desired pillar dimensions.

7) To improve pillar stability, dewatering in the area of the pillars and cable bolting (to maintain a tight mass and prevent ravelling) should be considered.

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Table 1. Approximate strength criteria for intact rock and jointed rock masses (9).

Rock quality (1)	Carbonate rocks with well developed crystal cleavage (dolomite, limestone and marble) (2)	Lithified argillaceous rocks, (mudstone, siltstone, shale and slate) (3)	Arenaceous rocks with strong crystals and poorly developed crystal cleavage (sandstone and quartzite) (4)	Fine grained polymineralic igneous crystalline rocks, (andesite, dolerite, diabase and rhyolite) (5)	Coarse grained polymineralic igneous and metamorphic crystalline rocks (amphibolite, gabbro, gneiss, granite, norite and quartz-diorite) (6)
Intact rock samples – laboratory size rock specimens free from structural defects (CSIR rating 100+; NGI rating 500)	$\sigma_{1n} = \sigma_{3n} + \sqrt{7\sigma_{3n} + 1.0}$ $\tau_n = 0.816(\sigma_n + 0.140)^{0.658}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{10\sigma_{3n} + 1.0}$ $\tau_n = 0.918(\sigma_n + 0.099)^{0.677}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{15\sigma_{3n} + 1.0}$ $\tau_n = 1.044(\sigma_n + 0.067)^{0.692}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{17\sigma_{3n} + 1.0}$ $\tau_n = 1.086(\sigma_n + 0.059)^{0.696}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{25\sigma_{3n} + 1.0}$ $\tau_n = 1.220(\sigma_n + 0.040)^{0.708}$
Very good quality rock mass – tightly interlocking undisturbed rock with unweathered joints spaced at 3 m ± (CSIR rating 85: NGI rating 100)	$\sigma_{1n} = \sigma_{3n} + \sqrt{3.5\sigma_{3n} + 0.1}$ $\tau_n = 0.651(\sigma_n + 0.028)^{0.679}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{5\sigma_{3n} + 0.1}$ $\tau_n = 0.739(\sigma_n + 0.020)^{0.698}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{7.5\sigma_{3n} + 0.1}$ $\tau_n = 0.848(\sigma_n + 0.013)^{0.702}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{8.5\sigma_{3n} + 0.1}$ $\tau_n = 0.883(\sigma_n + 0.012)^{0.705}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{12.5\sigma_{3n} + 0.1}$ $\tau_n = 0.998(\sigma_n + 0.008)^{0.713}$
Good quality rock mass – fresh to slightly weathered rock, slightly disturbed with joints spaced at 1–3 m (CSIR rating 65; NGI rating 10)	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.7\sigma_{3n} + 0.004}$ $\tau_n = 0.369(\sigma_n + 0.006)^{0.669}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{1.0\sigma_{3n} + 0.004}$ $\tau_n = 0.427(\sigma_n + 0.004)^{0.683}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{1.5\sigma_{3n} + 0.004}$ $\tau_n = 0.501(\sigma_n + 0.003)^{0.695}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{1.7\sigma_{3n} + 0.004}$ $\tau_n = 0.525(\sigma_n + 0.002)^{0.698}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{2.5\sigma_{3n} + 0.004}$ $\tau_n = 0.603(\sigma_n + 0.002)^{0.707}$
Fair quality rock mass – several sets of moderately weathered joints spaced at 0.3–1 m (CSIR rating 44; NGI rating 1.0)	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.14\sigma_{3n} + 0.0001}$ $\tau_n = 0.198(\sigma_n + 0.0007)^{0.662}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.20\sigma_{3n} + 0.0001}$ $\tau_n = 0.234(\sigma_n + 0.0005)^{0.675}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.30\sigma_{3n} + 0.0001}$ $\tau_n = 0.280(\sigma_n + 0.0003)^{0.688}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.34\sigma_{3n} + 0.0001}$ $\tau_n = 0.295(\sigma_n + 0.0003)^{0.691}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.50\sigma_{3n} + 0.0001}$ $\tau_n = 0.346(\sigma_n + 0.0002)^{0.700}$
Poor quality rock mass – numerous weathered joints spaced at 30–500 mm with some gouge filling/clean waste rock (CSIR rating 23; NGI rating 0.1)	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.04\sigma_{3n} + 0.00001}$ $\tau_n = 0.115(\sigma_n + 0.0002)^{0.646}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.05\sigma_{3n} + 0.00001}$ $\tau_n = 0.129(\sigma_n + 0.0002)^{0.655}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.08\sigma_{3n} + 0.00001}$ $\tau_n = 0.162(\sigma_n + 0.0001)^{0.672}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.09\sigma_{3n} + 0.00001}$ $\tau_n = 0.172(\sigma_n + 0.0001)^{0.676}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.13\sigma_{3n} + 0.00001}$ $\tau_n = 0.203(\sigma_n + 0.0001)^{0.688}$
Very poor quality rock mass – numerous heavily weathered joints spaced less than 50 mm with gouge filling/waste rock with fines (CSIR rating 3; NGI rating 0.01)	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.007\sigma_{3n} + 0}$ $\tau_n = 0.042(\sigma_n)^{0.536}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.010\sigma_{3n} + 0}$ $\tau_n = 0.050(\sigma_n)^{0.539}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.015\sigma_{3n} + 0}$ $\tau_n = 0.061(\sigma_n)^{0.546}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.017\sigma_{3n} + 0}$ $\tau_n = 0.065(\sigma_n)^{0.548}$	$\sigma_{1n} = \sigma_{3n} + \sqrt{0.025\sigma_{3n} + 0}$ $\tau_n = 0.078(\sigma_n)^{0.556}$

Figure 1. Holt-McDermott mine location.

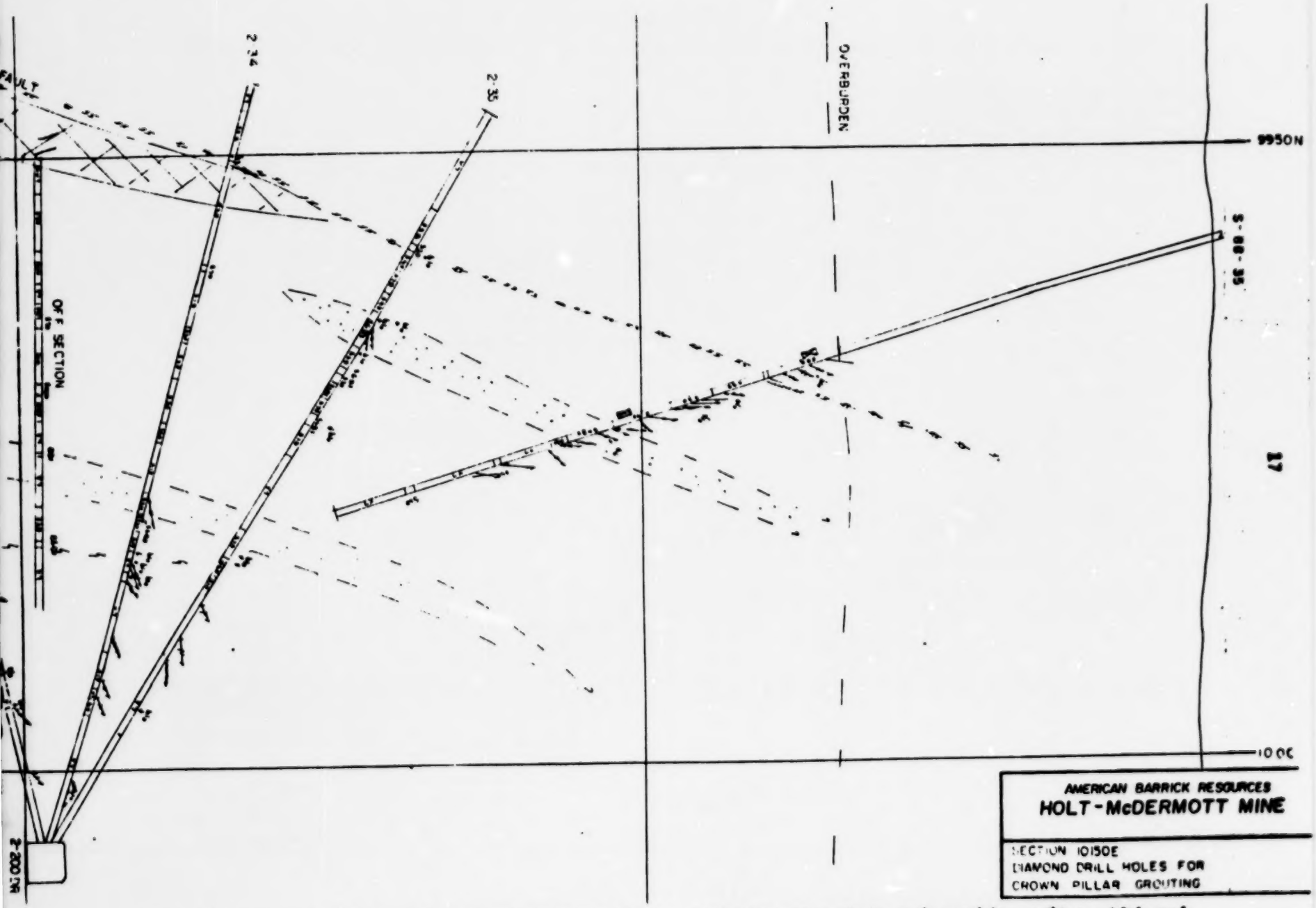

Figure 2. Transverse section showing the McKenna Fault, variation of orebody and graphite schist width and overburden thickness. Scale 1:400.

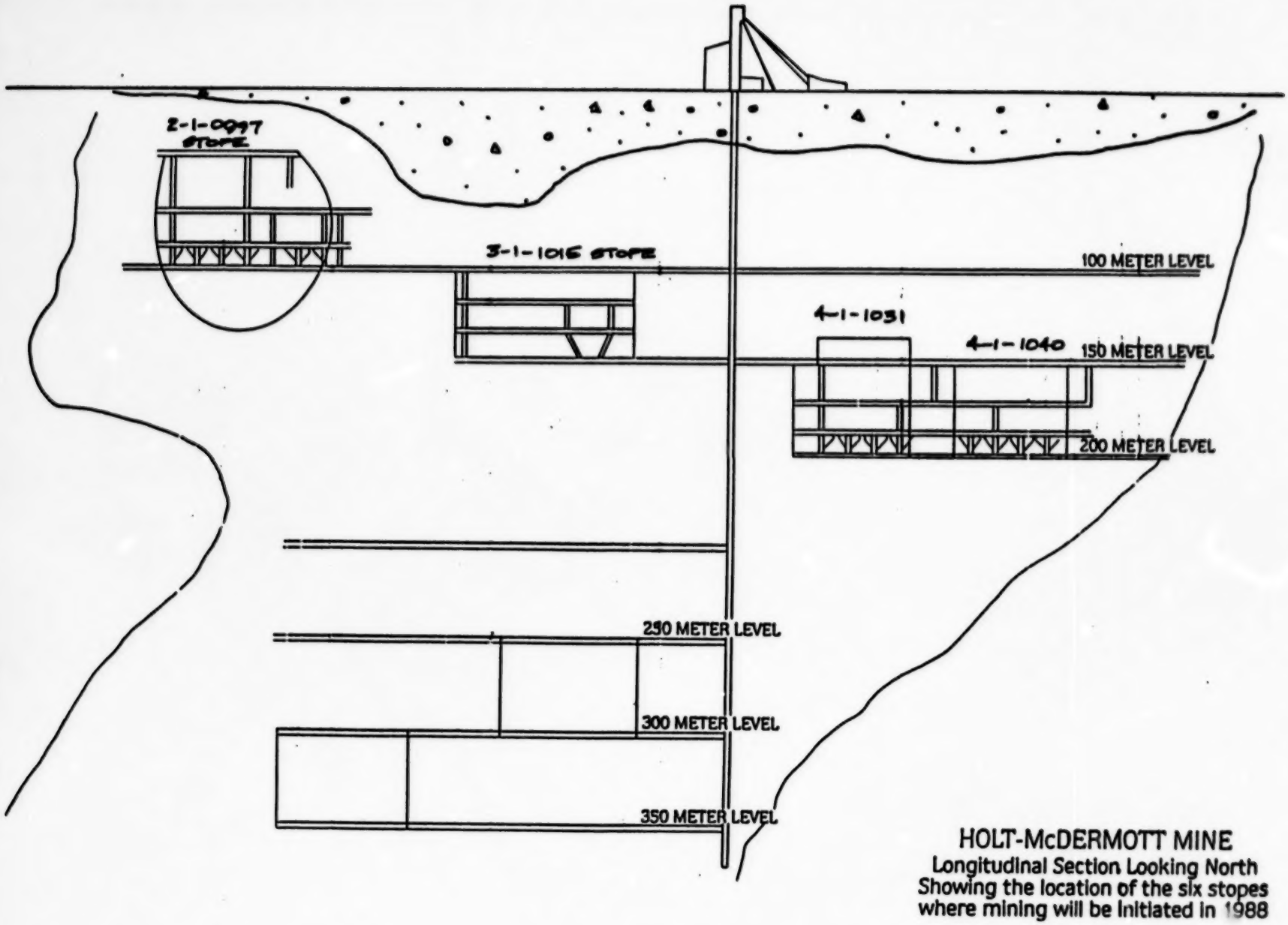

Figure 3. Longitudinal mine section showing variation of overburden thickness, near surface stopes and mine development to date. Scale 1:3000.

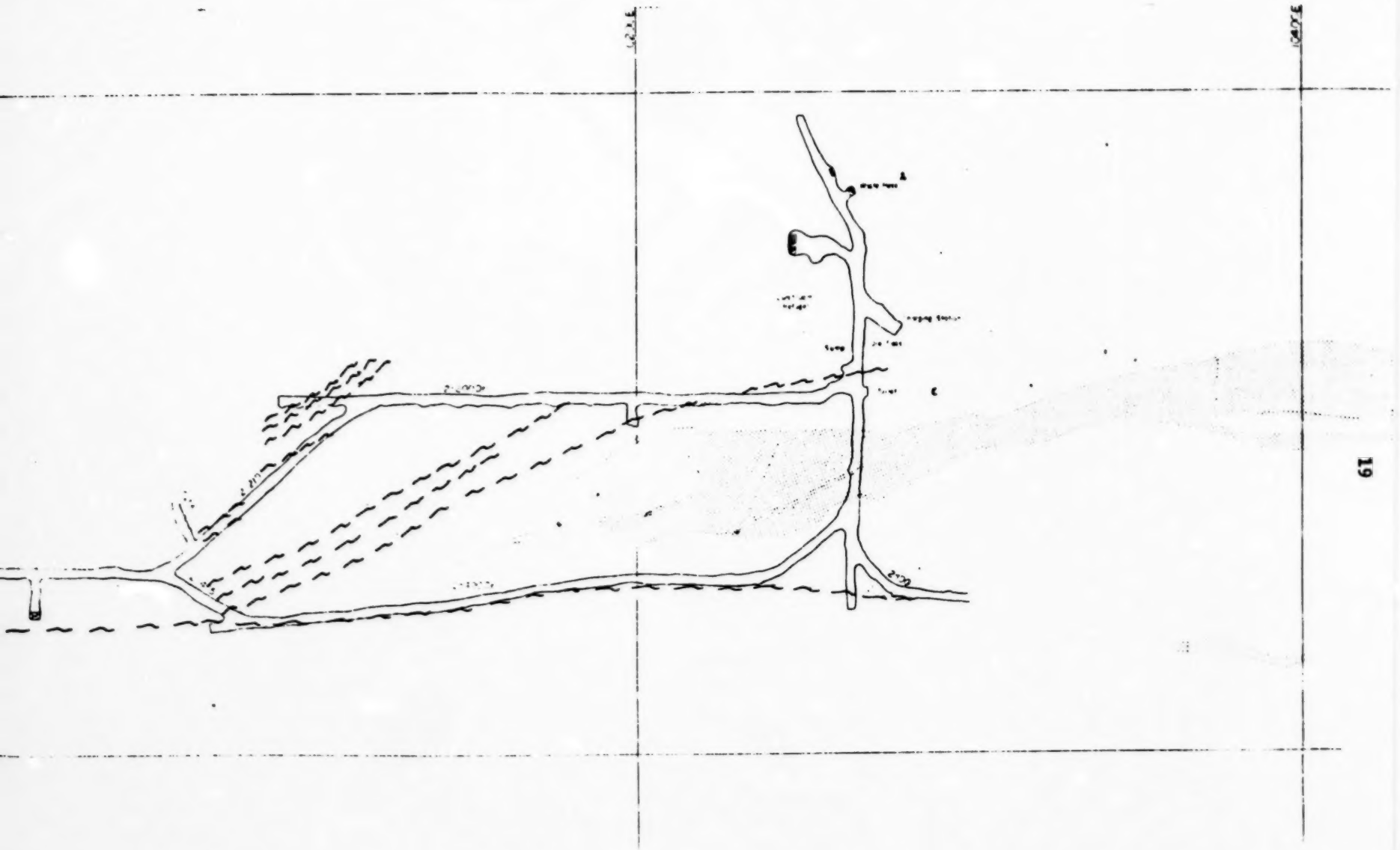

Figure 4. Mine plan showing cross faults and main fault, and the extent of the graphite schist zone. Scale 1:1500.

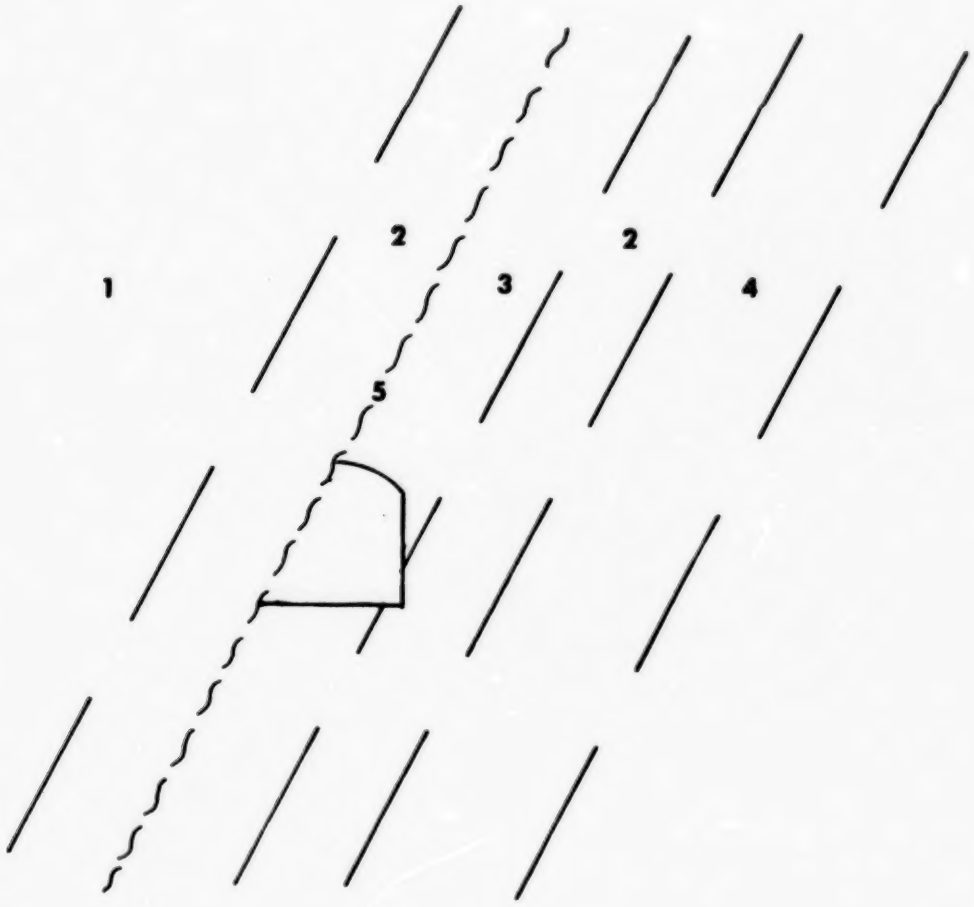

Figure 5. Example of classification into geomechanical sectors, Holt-McDermott mine. 1 far field hanging wall; 2 immediate hanging / foot wall; 3 ore zone; 4 graphite schist zone; 5 fault.

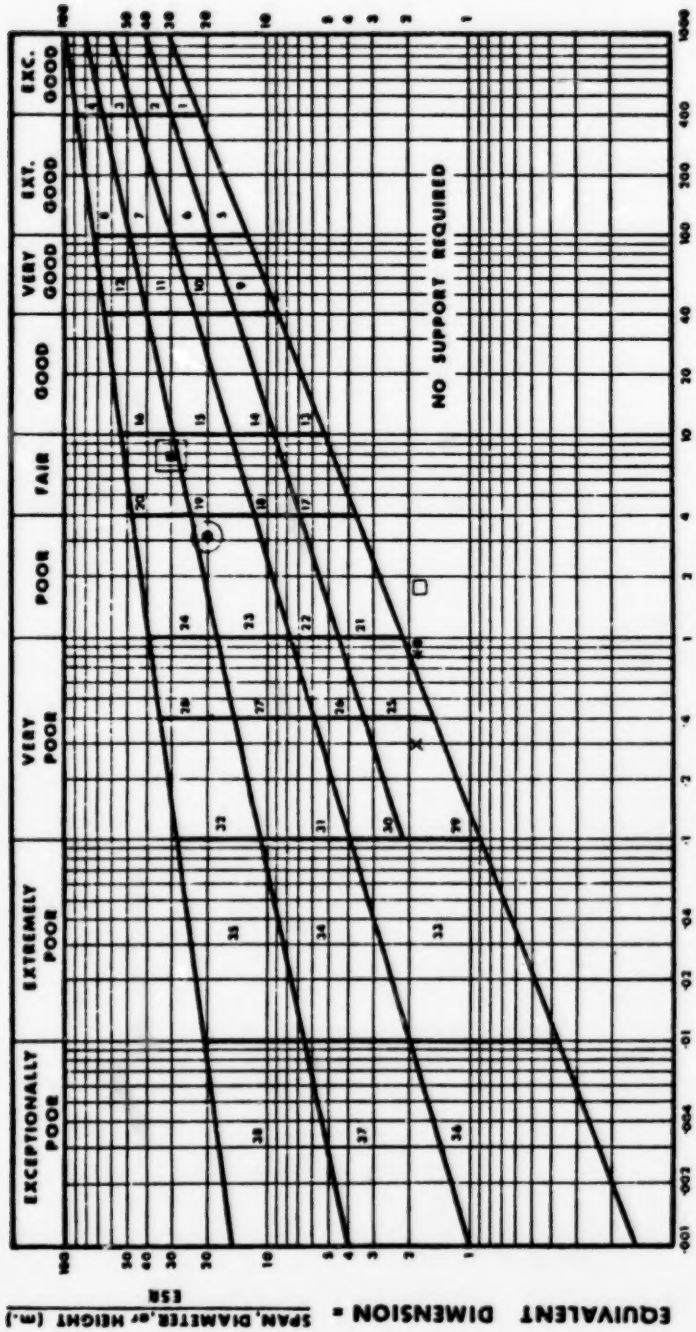

Figure 6. Qualitative representation of the Q values against opening specifications (ESR=1.6, permanent opening; drift span=3 m). × graphite schist zone; •immediate hanging wall/foot wall; •ore zone; □ far field hanging wall.

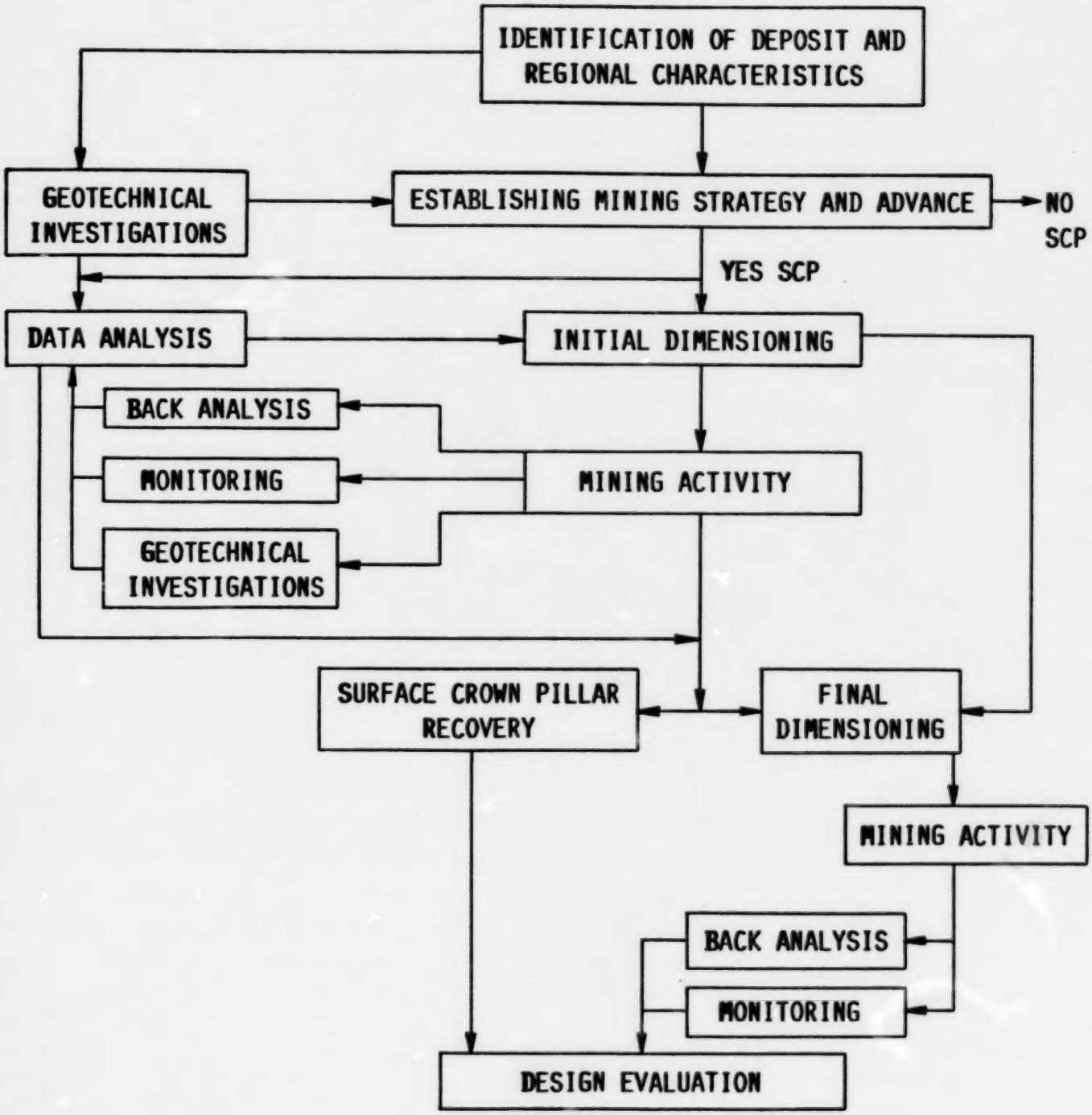

Figure 7. Simplified flow chart of surface crown pillar design (5).

APPENDIX 1

Calculation of Shear Failures and Plug-Like Failures Using Pre-Existing Stresses

Shear Failure In The Mass

By comparing the shear resistance, obtained from Hoek and Brown's (6) rock mass strength criteria, and induced shear stress, using in-situ stress conditions, a qualitative assessment of this situation can be obtained.

The following process indicates the application of this method. The values used are estimates. The mine operators are encouraged to perform valid tensile and uniaxial and triaxial tests and field (or core) rock mass characterization to obtain field Hoek and Brown strength parameters for geomechanical analyses as well as for future numerical modelling the mine wishes to carry out.

Expressed in terms of shear strength, the Hoek and Brown criterion is:

$$\tau_N = A\,(\sigma_N - \sigma_{tN})^B$$

where $\tau_N = \tau/\sigma_c$

$\sigma_N = \sigma/\sigma_c$

$\sigma_{tN} = [\,\tfrac{1}{2}m - (m^2 + 4s)^{\frac{1}{2}}]$

σ_c = unconfined compressive strength

m,s = field rock mass parameter obtained from lab tests (m_i, s_i) and transformed to field conditions;

A,B = constants, depending on the value of m.

Approximate strength relationships using this failure criterion, Table 1, provide this Mohr-Coulomb relationship:

$$\tau_n = 0.295\ (\sigma_n - 0.003)^{0.691}$$

Stresses in the Abitibi Greenstone Belt (7)(8) have been shown to have a ratio of horizontal to vertical stress of 1.7, in the N-S direction.

For the purpose of this preliminary assessment the effect of the fault on stress distribution is not considered.

The intermediate can be taken as 1.7 times vertical stress. The stresses at the base of a 40 m surface crown pillar are (Figure A1-1):

$$\sigma_v = \frac{0.026}{\text{meter}} \text{ X } 40 \text{ m} = \text{MPa}$$

$$\sigma_H = 1.7 \text{ X } 1.04 = 1.77 \text{ MPa}$$

Figure A1-2 shows the Mohr circle for this stress condition as well as the Mohr-Coulomb strength envelope for the adopted Hoek and Brown criterion values.

Because the stress circle does not intersect the strength envelope, shear failure from stress is not expected. But the rock mass must be effectively broken, with long discontinuities oriented parallel to shear planes, in order for this condition to occur. The jointing of the rock mass is not suitable enough for this, even though some ravelling areas might be conducive to this situation.

Plug-Like Failure

Plug-like failure is worthwhile considering, especially when the graphite zone adjoins the stope. A rock volume between the hanging wall fault and this zone could slip into the opening, Figure A1-3.

The stresses acting on the sliding rock mass are:

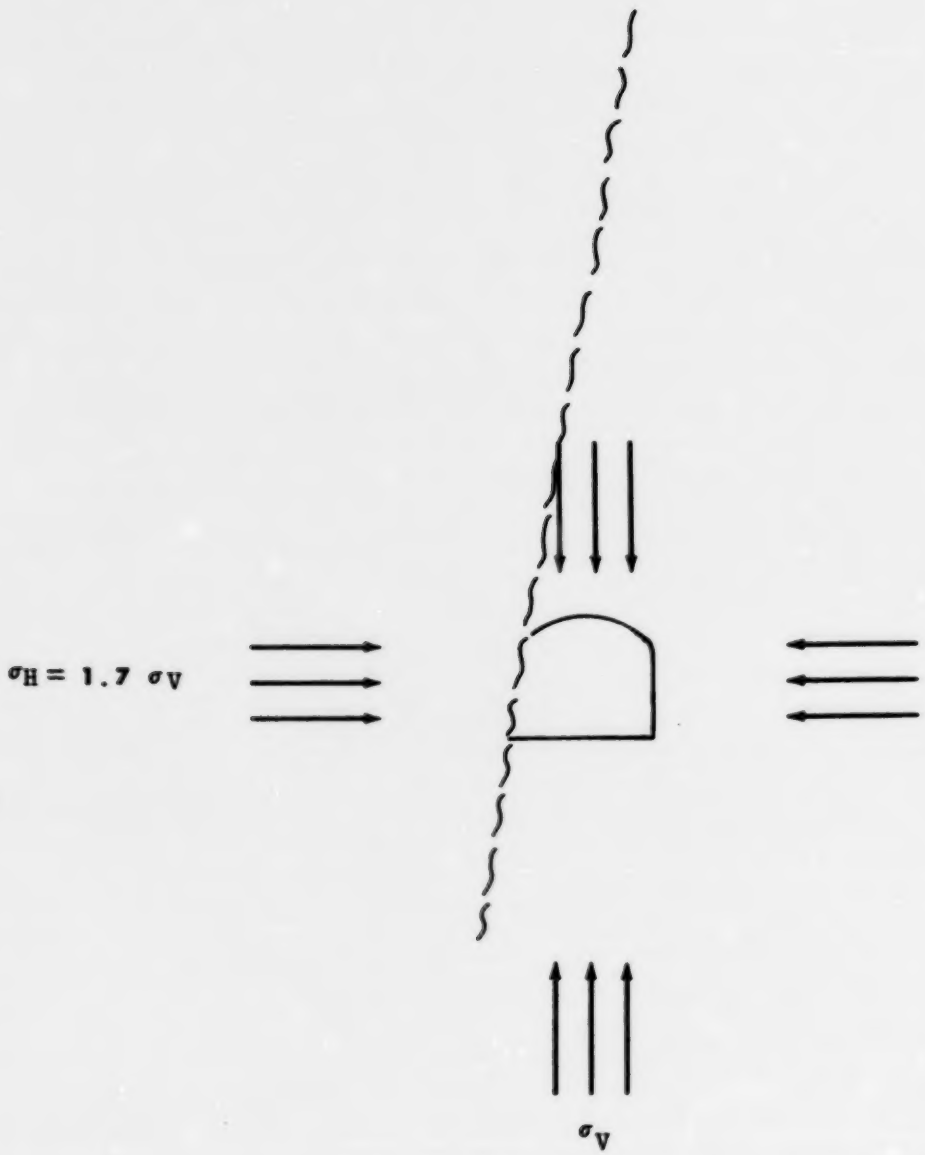

Figure A1-1. In-situ stresses imposed on a mine transverse section (not including fault effects).

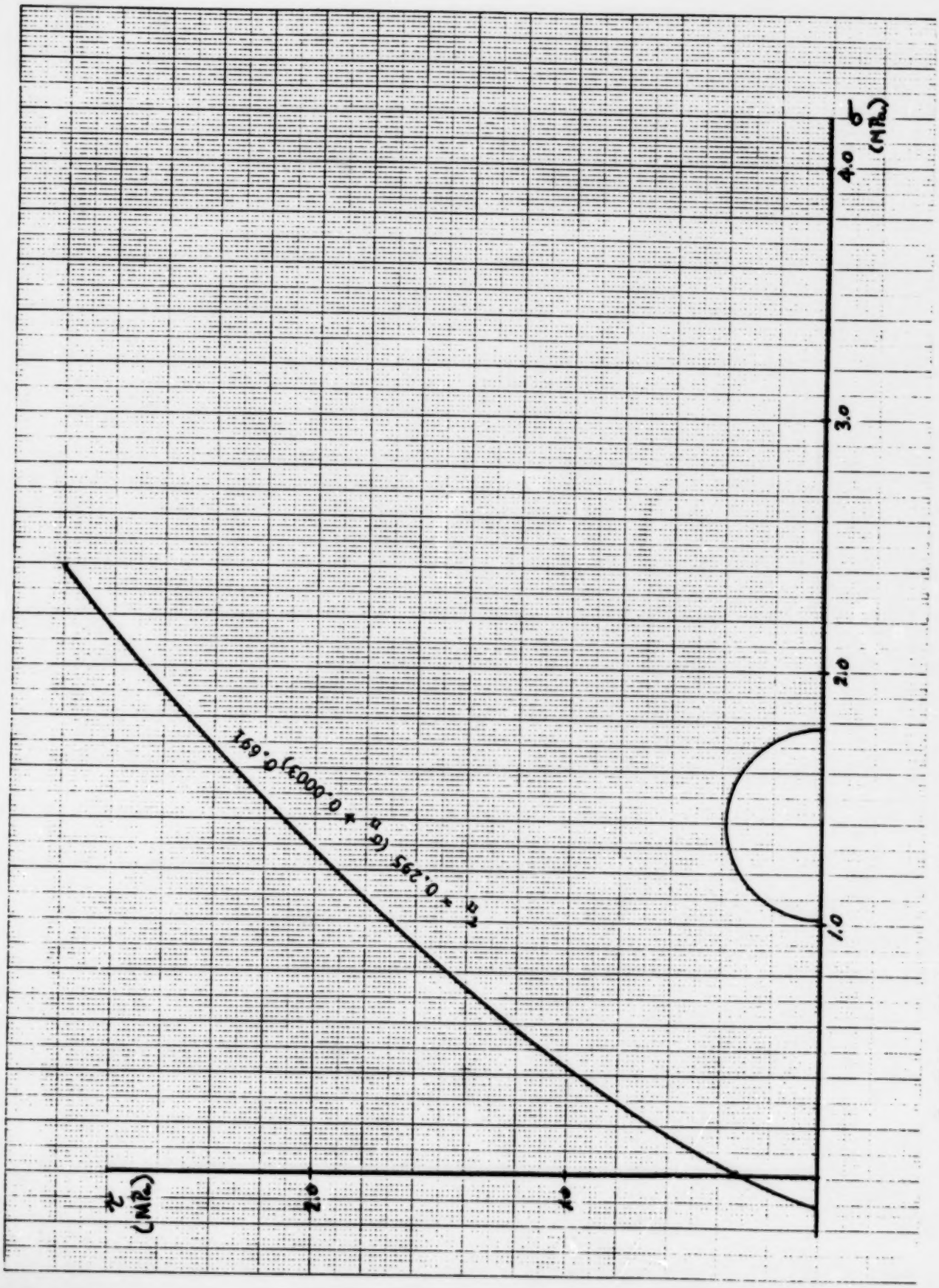

Figure A1-2. Hoek and Brown failure envelope and in-situ stress conditions.

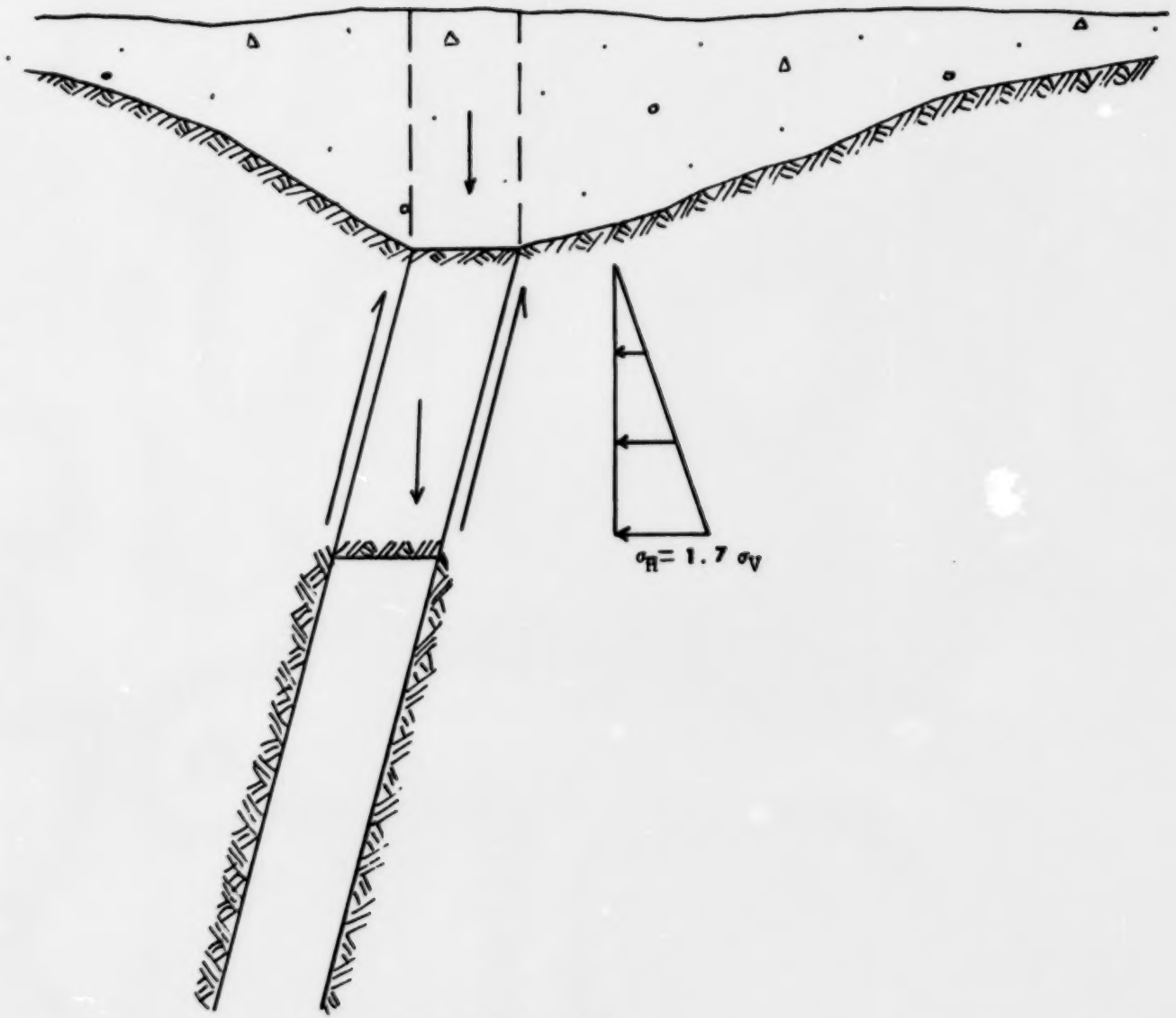

Figure A1-3. Force diagram, plug-like failure.

- overburden weight;
- weight of rock volume between the failure planes;
- hanging wall friction;
- footwall friction;
- lateral stresses.

The analysis will compare stabilizing forces (shear strength) and destabilizing forces (weight of soil and rock), per unit thickness in the longitudinal direction.

The driving (destabilizing) forces are the soil and rock weight, parallel to footwall plane of sliding:

$$\left[\left(0.022 \frac{MN}{m^3} \times 4m \times 30m\right) + \left(0.026 \frac{MN}{m^3} \times 4m \times 40m\right) \right] \cos 20°$$

The stabilizing forces are friction resistance of the footwall (rock + soil normal weight and in-situ stress imposed) as well as the hanging wall (in-situ stress) slip surfaces (σ_n F.W. tan Ø F.W. + σ_n H.W. tan Ø H.W.):

$$\tan Ø \text{ F.W.} \left[\left[\left(0.022 \frac{MN}{m^3} \times 4m \times 30m\right) + \left(0.026 \frac{MN}{m^3} \times 4m \times 40m\right) \right] \sin 20° \right.$$

$$\left. + 1.7 \left(0.026 \frac{MN}{m}\right) (40m) \left(\frac{2}{3}\right) (\cos 20) \right] +$$

$$\tan Ø \text{ H.W.} \left[1.7 \left(0.026 \frac{MN}{m}\right) (40m) \left(\frac{2}{3}\right) (\cos 20) \right]$$

In a F_s ratio, with a Ø = 20° for the gouge filled hanging wall surface Ø = 15° for footwall graphite schist (3):

$$F_s = \frac{\text{stabilizing}}{\text{destabilizing}}$$

$$F_s = \frac{\tan (15) (3.44) + \tan (20) (1.11)}{6.39} = 0.21$$

This analysis does not take into account third dimension rock

behaviour, but what it does show is that even if there is a higher Ø value for either slip plane the F_s under natural stress conditions will still be unacceptable.

Creating a stope with the immediate footwall composed of the basaltic mass reduces substantially the tendency to slip.

APPENDIX 2

Calculation of Stress Redistribution Around A Shallow Opening

The elastic formulation of Geldart and Udd to calculate tangential stress around an opening is:

$$\frac{\sigma_\eta}{S_y} = \frac{2\nu(1+\kappa) + (1-\kappa)(1-\nu^2)\cos\beta + (1-\kappa)(1+\nu)^2\cos 2(\beta-\eta)}{(1+\nu^2) + (1-\nu^2)\cos 2\eta}$$

σ_η = tangential stress on the ellipse surface
ν = semi-major axis / semi-minor axis = axis ratio
κ = S_x/S_y = pre-mining horizontal / vertical stress = stress ratio
β = angle clockwise from semi-major axis to S_x
η = elliptical co-ordinate measurement clockwise from the semi-major axis.

Based on stress measurements in the Abitibi-Greenstone Belt (7)(8), and planed stope size under surface crown pillars, Figure A-2.1:

ν = 25/2.5 = 10
κ = 1.7
β = 70°

At the bottom, centre, of the surface crown pillar ($\eta = 0°$) the tangential stress is:

$\sigma_\eta = S_y\,(71.4)$

= 1.77 MPa (71.4) = 126.4 MPa (compression)

At the upper stope corners ($\eta = 4°$):

$\sigma_\eta = S_y\,(44.8)$

= 1.77 MPa (44.8) = 79.3 MPa (compression).